4702

STATE OF CALIFORNIA
HIGHWAY TRANSPORTATION AGENCY
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

BRIDGE DEPARTMENT

July 1964

EPOXY BONDED AGGREGATES AS SHEAR CONNECTORS IN COMPOSITE CONSTRUCTION

Progress Report

Prepared in Cooperation with
The U.S. Department of Commerce,
Bureau of Public Roads

EPOXY BONDED AGGREGATES
AS SHEAR CONNECTORS
IN COMPOSITE CONSTRUCTION

Progress Report

Test Performed by:	Materials and Research Department
Report Written by:	Carl F. Stewart Associate Bridge Engineer
Directed by:	W. J. Jurkovich Senior Bridge Engineer
	John J. Kozak Supervising Bridge Engineer
General Supervision:	John L. Beaton Materials & Research Engineer
	James E. McMahon Asst. State Highway Engineer, Bridges
	J. C. Womack State Highway Engineer

64-32
DND

Epoxy Bonded Aggregates As Shear

Connectors In Composite Construction

SYNOPSIS

This report concludes that aggregates bonded to steel, shop applied, can serve as shear connectors in composite girder construction. Results of small sample exploratory tests and the first of four proposed prototype girder tests are included.

The prototype girder failed at 75% of its theoretical ultimate capacity. A 90% bond failure of epoxy to steel was the mode of failure. This bond to steel failure is contrary to the shear in concrete failure that generally resulted during the preliminary small sample tests. Application of epoxy to an unclean steel surface is believed to have been the most significant factor that contributed to the premature failure of the prototype girder.

SCOPE

The use of epoxy bonded aggregates to transfer shear between steel girders and the concrete deck slab in composite construction was proposed in 1961 by W. J. Jurkovich, Senior Bridge Engineer, California Division of Highways. The epoxy bonded aggregates would serve as shear connectors only. A sufficient number of conventionally welded steel connectors would be added to prevent the vertical separation of slab and girder. The epoxy and aggregates would be shop applied.

The purpose of employing epoxy-aggregates as a connecting medium in association with the conventional welded steel connector is to reduce the concrete deck surface cracking that is inherent in steel girder structures, to eliminate welded shear connectors entirely in areas of negative moment, and to gain economy in composite construction. The validity of this reasoning is subject to verification and can only be answered by placing structures of this type in service. However, before this can be done a more important question of whether epoxy bonded aggregate could adequately transfer shear between slab and girder must be answered.

A test was initiated to check the validity of the premise that aggregates could act as shear connectors in composite construction.

PRELIMINARY TESTS

Push-out

Push-out tests were conducted on 23 samples of concrete that had been cast between 2 steel plates. Shear resistors in the form of epoxy bonded aggregate, along with various combinations of simulated mechanical hold downs, had been attached to the steel plates.

The first group of specimens tested by the push-out method employed 1", 3/4" and 1/2" aggregate sizes without hold down devices. There was a noticeable difference in the mode of failure between the various specimens. A failure of the 1" aggregate specimens was usually by shear in the aggregate. Failure was always in the concrete of the 1/2" specimens. The 3/4" had both type of failures. The 512 psi average shear resisting value of the 1/2" aggregate specimens was higher than the average values of the other two sizes. From the result of these tests it appeared the 1/2" average size aggregate was an optimum size and it was therefore adopted as the size to be used in all future testing.

Accelerated

An accelerated corrosion durability test was conducted on six 12" x 11" plate specimens. A thixotropic epoxy was placed on plates in the following thickness: 40 mils, 1/8" and 1/4". One-half inch average size crushed aggregate was broadcast over the wet epoxy and then pushed into it. Three weeks after the epoxy application the specimens were placed in a 5% solution salt spray bath. (The only significance in the three weeks delay in placing the specimens in the salt bath was the availability of the bath.) The specimens were subjected to 488 hours of salt spray exposure. This was followed by 12 cycles of -10° to 78F° freeze-thaw cycles. These specimens were then returned to the salt spray bath for an additional 500 hours. Concrete was cast between the companion plates and they were subjected to the push-out test method. The average shear for the 3 samples was 280 psi.

The unprotected portion of the steel plates were badly corroded after the salt spray exposure. The entire unprotected area was covered with flakes of rust. The condition of the steel under the epoxy was checked after the push-out tests. There were a few areas of dark stains or discoloration, which could have been early stage oxidation, but no flakes of rust or heavy rust could be found that was similar to that found on the unprotected steel. The dark stains were generally within 1/8" of the edge of epoxy or under epoxy that had been made quite thin by pieces of aggregate having either been pushed completely through or almost through the epoxy. The discolored areas were a very small percent of the total area covered by epoxy.

The stiff epoxy used in this experiment did not have sufficient reserve liquid to "wet" the aggregate that was pushed into it. Poor adhesion of the epoxy to aggregate was the general mode of failure during the push-out tests. This was especially true for the 40 mil thickness of epoxy.

PROTOTYPE TESTS

Prototype Girders

Based on the results of the preliminary test, previous experiences with epoxy, and experiences by others [1, 2] with epoxy in similar composite tests, it was decided to advance into a prototype test project. The project was initiated for the testing of the following prototype composite girders:

(1) A 16WF71 40-foot long steel beam with 7" deep by 34" wide concrete slab made composite with the steel by a single row of 3/4" diameter welded studs at 24" c-c for full length of beam and 1/2"$\pm$ aggregate bonded to top flange with epoxy over 13' at each end of beam. (Figure 1)

(2) Same as (1) but use welded studs at 48" c-c for full length of beam.

(3) Same as (1) but eliminate studs and place epoxy bonded aggregate for full length of beam.

(4) Same as (1) but eliminate epoxy and aggregate.

A single predetermined load is to be applied in controlled increments at 4 feet from one end of the girder. This load is to be replaced by two symmetrical loads placed in controlled increments at approximately the 1/3 points until the girder fails.

During the loading of these girders the following data is to be collected:

(1) Stresses in the flanges and web of the steel beam and top and bottom surfaces of the concrete slab at the applied load.

(2) The movement of concrete deck relative to the steel beam at the end 12' of the unit.

(3) The deflection of the girder unit at the 1/4 and 1/2 points. (Figure 2)

Test Facilities

Two parallel 33WF130 girders, supported at their ends on reinforced concrete pedestals, were to serve both as a support for the test girders and as a kicker for the jacking yokes, through which the prescribed loads were to be applied to the test girder.

Construction of the facilities for conducting the girder tests were completed in August, 1963, and the first of the four proposed test girders was constructed soon thereafter. (Figure 3)

PROTOTYPE GIRDER NO. 1

Epoxy-Aggregate

The epoxy used on the first test girder had the same basic components as the epoxy generally used by the California Division of Highways for bonding new concrete to old. Following is this epoxy formulation:

Component A	Percent by Volume
Epoxy Resin[1]	50
Colloidal Silica (loose volume)	50
Component B	
Polysulfide Polymer[2]	82
2, 4, 6 - Tri (dimethylaminomethyl) Phenol[3]	18

1. Viscosity, 5-9 poise at 25°C, Epoxide equivalent 175-205. Manufactured from epichlorohydrin and bisphenol A.

2. Specific Gravity, 1.27 at 20°C; viscosity, 700-1200 centipoise at 25°C; pH water extract, 6.0-8.0; moisture content, 0.1% maximum; pour point, -15°F; average molecular weight, 1000. A disfunctional mercaptan made from 98 mole percent of bis (2-chloroethyl) formula and 2 mole percent of trichloropropane.

3. Formula weight 265; specific gravity at 25°/25°, 0.973; refractive index, 1.514 at 25°C; distillation range 96% at 130-160°C (0.5-1.5 mm); flash point (Tag open cup), 300°F, water content, 0.06% maximum.

Mix: 2 parts by volume of Component A with one part by volume of Component B.

The aggregate was crushed and screened from meta volcanic material obtained from the Mission Valley Area of California and fell within the following gradation.

Sieve Sizes	Maximum % Passing Sieves
3/4"	100
1/2"	40-60
3/8"	0

The epoxy was spread to a thickness of 1/16" on the sand blasted steel surface. Ambient temperature at time of application was approximately 100°F. An excess of aggregate was broadcast onto the epoxy. After the epoxy had cured, all loose aggregate was removed from the girder by air blasting. (Figure 4)

Deck Slab

A concrete deck slab was poured 7 days after the epoxy-aggregate application. The concrete was a six sack, 1-1/2 maximum size aggregate mix with 1-1/2 - 2" slump. The exposed surface was sprayed with a curing compound. The average 28 days compressive cylinder strength of 3 samples was 4800 psi.

Instrumentation

To measure strains in the units, SR-4 gages were attached to the steel and A9 gages were attached to the concrete. 0.0001" Ames dials were attached to the steel to measure slip between steel and concrete. A layout of instrumentation is shown on Figure 2.

Loading

Tests were begun on the girder 28 days after the slab was poured.

Loading of the girder was accomplished through hydraulic jacks placed between the top of girder and steel frame jacking yokes that were supported by the parallel support girders. Load cells were placed between the jacks and jacking yokes for accurate load applications.

The first loading sequence was made with a jack placed 4' from one support. A 1 kip load was applied and was considered a zero load for calibration. The load was increased to 65 kips in 5 kip increments.

The load was decreased to 1 kip and 65 kips was re-applied in 10 kip increments. The load was again reduced to 1 kip and reapplied in 20 kip increments. After each increment loading strain, slip and deflection readings were taken.

The second loading sequence was made with jacks placed 11' from each support. A 1 kip load was applied as a zero load to each jack. The load was increased to 30 kips in 5 kip increments. It was reduced to 1 kip and again applied in 10 kip increments. It was again reduced to 1 kip and again re-applied in 10 kip increments until 30 was reached, at which time it was continued at 5 kip increments to failure. Strain, slip and deflection readings were again taken at each increment of loading. See Figure 5 for a plot of strain readings taken during the last loading sequence.

RESULTS

During the application of loads for the loading sequence of 65 kips at 4 feet from one support it was noticed that some slip between concrete and steel continued for several minutes after the higher loads had been placed. Upon completion of the sequence loading a static load of 65 kips was placed on the girder and slip gage readings were taken every 15 minutes over a 2-hour period. The purpose of these readings was to determine at what rate the slip would continue. A plot of the data for four gages is shown on Figure 6. There was a very small difference in the readings of the gages not shown. It is seen from these data that slip would have continued, at a decreasing rate, for some time.

During the symmetrical sequence loading the need for additional shims under the jacks caused the two 40 kip loads to be retained for 45 minutes before they were increased to 45 kips. After 30 minutes of this delay the load cells indicated a loss of loading on the girder. The loads were again brought up to 40 kips and another set of readings were taken. Slip action is again reflected in the Strain Distribution, Load-Slip, and Load-Deflection curves (Figures 5, 7 & 8). The loading increments were generally placed at about 5 minute intervals.

The last set of readings taken during the symmetrical sequence loading that could be considered as having reasonable accuracy appears to be the first readings for the 40 kip loads. At this loading the Strain-Distribution curves reflect a reasonable amount of continuity between the slab and girder. A certain amount of discontinuity due to slip is reflected in both the 30 and first 40 kip loads, but for comparative purposes this amount of slip can be ignored and complete continuity can be assumed when calculating stresses. However, the magnitude of discontinuity for the second set of 40 kip readings is obviously too great to assume complete interaction. For these and higher loads the steel and concrete sections are approaching a condition of two independent beams. The results show that for two sustained 40 kip loads the slip between the concrete slab and steel beam was so rapid that an approximate 30 percent change in results would be reflected in a 10 minute delay in taking the readings.

Loading was discontinued after 3 steel studs had sheared at one end of the girder and the concrete had slipped about 1/4 of an inch. (This occurred at the opposite end from where the first sequence loading has been placed.) There was over 90% bond failure of epoxy to steel at this end and upon removing a portion of the slab it was found that the bond of epoxy to aggregate was also poor. (Figure 10) Maximum slippage of 0.0338" occurred at Gage 7 at the other end of the girder. None of the steel connectors sheared at this end. It was difficult to determine the mode of epoxy failure at this end, but it appeared to be similar to the opposite end failure. There was a 4.9" residual deflection at the center span of girder after the loading jacks were removed. The failure occurred gradually

DISCUSSION

By assuming complete interaction and failure at the two 40 kip loads, which are approximately 75% of the anticipated failure loads, the calculated horizontal shear between slab and girder at failure was 240 psi. This is less than 1/2 the anticipated epoxy shear resisting value when acting independently of the steel studs. Removal of the slab after the test revealed a 90% bond failure of epoxy to steel therefore, insufficient bond rather than shear appears to have been the primary cause for the prototype failure. Failure by bond is contrary to the aggregate or concrete mode of failure that occurred in the preliminary push-out tests. The reason for this unsuspected type of failure caused concern, and additional sample tests were initiated in an effort to establish contributing factors. Some factors that were suspected as possible contributors to the failure were:

(1) Incorrect mixture of the epoxy components.

(2) Adverse plastic flow characteristics of epoxy.

(3) Application of epoxy to a hot steel surface.

(4) Application of epoxy to an unclean steel surface.

(5) Differences in coefficient of thermal expansion between epoxy and steel and concrete.

The tests were aimed at exploring what effect these factors would have on the bond strength of epoxy.

Mixture

The first approach was to examine samples of the failed epoxy. Samples of the epoxy were removed from the slab less than 24 hours after the test. These samples were compared with samples made from an epoxy that duplicated the prototype admixture. The primary purpose of this comparison being to ascertain if the correct proportions of mixtures had been combined, both in the original formulation and the final mixing. No discrepancies were found in the mix proportions.

The only apparent differences between the samples was in their color and flexibility. The samples from the prototype test were much more flexible and possessed a deeper red color than did the control samples. The deeper red color was not unexpected because the epoxy was placed on the steel girder when the girder was over 100°F, whereas the control sample was cured at approximately 78°F. Past experiences with the formulation used has been that the higher the curing temperature the more its color approaches a rusty red.

The excessive flexibility of the prototype samples is, however, contrary to what is generally experienced, even at higher curing temperatures. The reason for this excessive flexibility has not been determined.

Plastic Flow-Heat Curing

The next approach was to conduct pull-out and double shear tests in an effort to determine what effect head-curing has on the shear and bond strength and the flow characteristics of epoxy. Since the primary purpose of these subsequent tests was for comparison of differentials in heat curing, outside variables were minimized by testing the samples under double shear conditions rather than the single shear condition that exists during prototype testing.

During these subsequent tests the performance of the prototype epoxy formulation (hereafter referred to as formulation A) was compared with a formulation that was used by Arizona in a similar composite beam test (hereafter referred to as formulation B), and a formulation that was a modified formulation B (hereafter referred to as formulation B^1). Formulation B^1 was a more viscous admixture than formulation B.

Double Shear

The double shear test was made by adhering a single strap of steel between two other similar steel straps with 1/16" thick layers of epoxy. The total area was 12 square inches (2 x 2" x 3"). After an 18 day curing period the straps were pulled apart. Eight samples involving 3 samples each of formulations B and B^1 and 2 samples of formulation A were tested. One sample of each formulation was cured at 78°F. The remaining samples were cured at 140°F after application of the epoxy to straps that had been preheated for one hour at 140°F. The results of this test are shown in Figure 9. The results show that a change in curing temperatures from 78"F to 140"F does not adversely affect the resisting value of epoxy in double shear. Formulation A compared favorably with the other formulations in total resisting value.

Pull-Out; Semi-Rapid

Formulations A & B[1] were used to bond aggregates to both sides of a 2" wide strap over a length of 6". (24 sq. inches of total area) One sample of each formulation was cured at 78°F and one at 140°F. The straps were cast in non-reinforced concrete blocks after the epoxy had cured and then were pulled from the concrete after it had obtained a strength of at least 4500 psi. The concrete block was supported by a 1" thick steel plate during the testing period. The steel plate had an opening in the center just large enough for passage, without binding, of the strap. The purpose of this being to offer as much support as possible to the concrete to prevent it from spalling or popping out. To obtain a better distribution of the load over the irregular surface of the concrete, a 1/2" premolded fiberboard was placed between the concrete block and support plate.

The first apparent failure during the pull-out test method was cracking in the concrete block. As additional load was applied the cracks widened until the block split. All concrete blocks split prior to failure of the epoxy. The fact the epoxy did not reach failure prevents a comparison of the variables. All blocks failed at approximately the same load and there was approximately 1000 psi shear on the epoxy area at failure.

Pull-Out; Sustained Loading

Four additional samples were made using the same principle as the semi-rapid pull-out test except the concrete blocks were reinforced with #5 wire coiled in a 6" diameter on a 1 1/2" pitch and a 1/2" thick rubber pad replaced the premolded fiberboard during testing. These samples are currently being subjected to a series of sustained loads for a minimum of 24 hours for each load. The first load causes a shear of 250 psi through the epoxy. The subsequent loads increase the shear in 250 psi increments until the epoxy is subject to 1000 psi shear for a minimum of 24 hours; one sample retained this load for 96 hours. Three samples have been tested thus far. The fourth one will be subjected to each load for a much longer period than were the first 3.

0.0001 inch increment Ames dials were positioned to record slip of the steel strap. There has not been any appreciable slip in any of the samples tested thus far.

After each sample had completed the cycle of sustained loads, the strap was pulled from the concrete in the same manner as in the semi-rapid test method. The mode of failure for this test was the same for all specimens. In each case there were a few small hairline cracks in the concrete that appeared prior to failure but did not appear to have widened substantially at the time of failure. All failures were due to bond failure of the epoxy to the steel. The three specimens tested thus far failed at an average value of 1500 psi shear on the epoxy area.

Thermal Coefficient

Problems involving differences in thermal coefficient of expansion of steel and epoxy are currently being studied. Tests concerned with this problem are to be initiated soon.

Application

The steel surface was sandblasted prior to the epoxy application. There was no attempt made to further clean the steel surface. Subsequently it has been realized that some sandblasting equipment occasionally discharges fine particles of oil along with the air and sand. It is conceivable that a thin film of oil was deposited on portions of the prototype steel girder prior to the epoxy application. If such were the case the bond between epoxy and steel would have been adversely affected. In retrospect, the top of the girder should have been cleaned with a solvent subsequent to sandblasting.

CONCLUSIONS

(1) Strain and slip curves indicate that good interaction between slab and girder was provided through the epoxy bonded aggregates up to approximately 75% of the ultimate capacity of the girder, or a horizontal shear of 240 psi in the epoxy. Under a sustained shear of 240 psi however, slippage between slab and girder occurred and interaction was lost.

(2) Insufficient bond of epoxy to steel due to an application oversight appears to be the most significant factor that contributed to the unexpected failure of the epoxy at the low shear value. Post exploratory tests concerning shear and plastic flow properties vindicated

the epoxy formulation employed. They showed the prototype formulation compared favorably with other formulations. None of the formulations were susceptible to plastic flow during double shear sustained load tests. The exploratory tests failed to reveal the reason for the prototype slip phenomenon at the relatively low shear stress.

(3) The results from the one prototype girder test, although not conclusive, are promising and indicate that epoxy bonded aggregates can be applied to serve as shear connectors in composite girder construction.

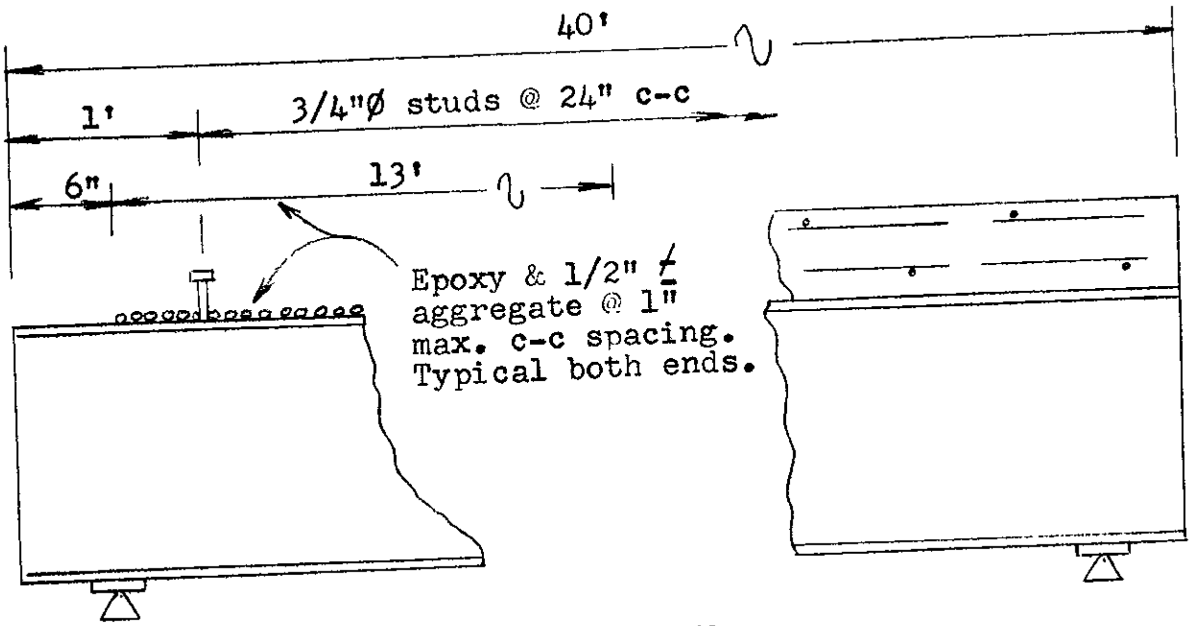
(4) Prototype tests should be continued. Conditions of Girder No. 1 regarding stud and epoxy aggregate layout should be retested.

REFERENCES

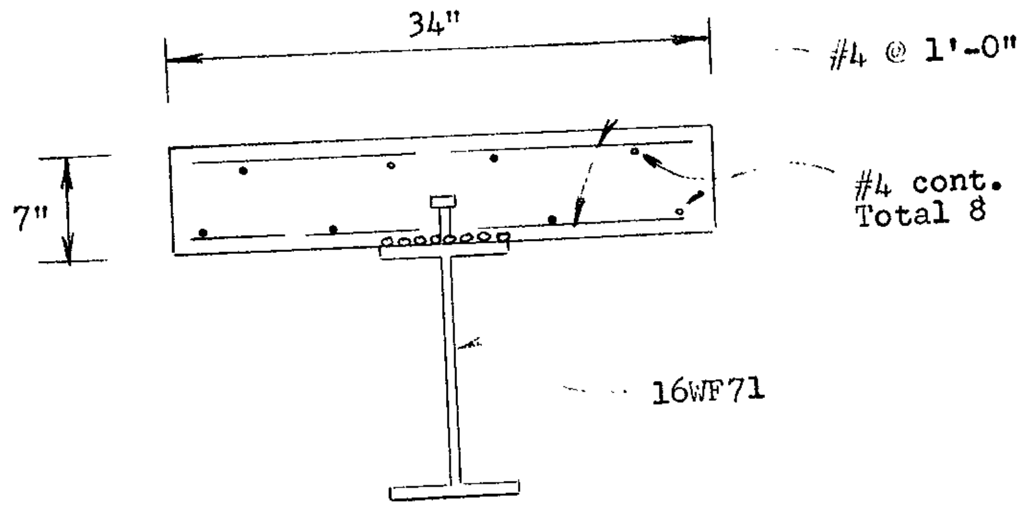
1. Miklofsky, H., Brown, M. Jr., and Gonsior, M., "Epoxy Bonding Compounds as Shear Connectors in Composite Beams". Interim Report, Physical Research Project No. 13, Dept. of Public Works, State of New York.

2. Kriegh, J., and Endelbrock, E. "The Use of Epoxy Resins in Reinforced Concrete - Static Load Tests". Final Progress Report to the Arizona Highway Department.

EPOXY-AGGREGATE
SHEAR CONNECTOR TEST



ELEVATION



CROSS SECTION

FIG. 1 PROTOTYPE GIRDER NO. 1

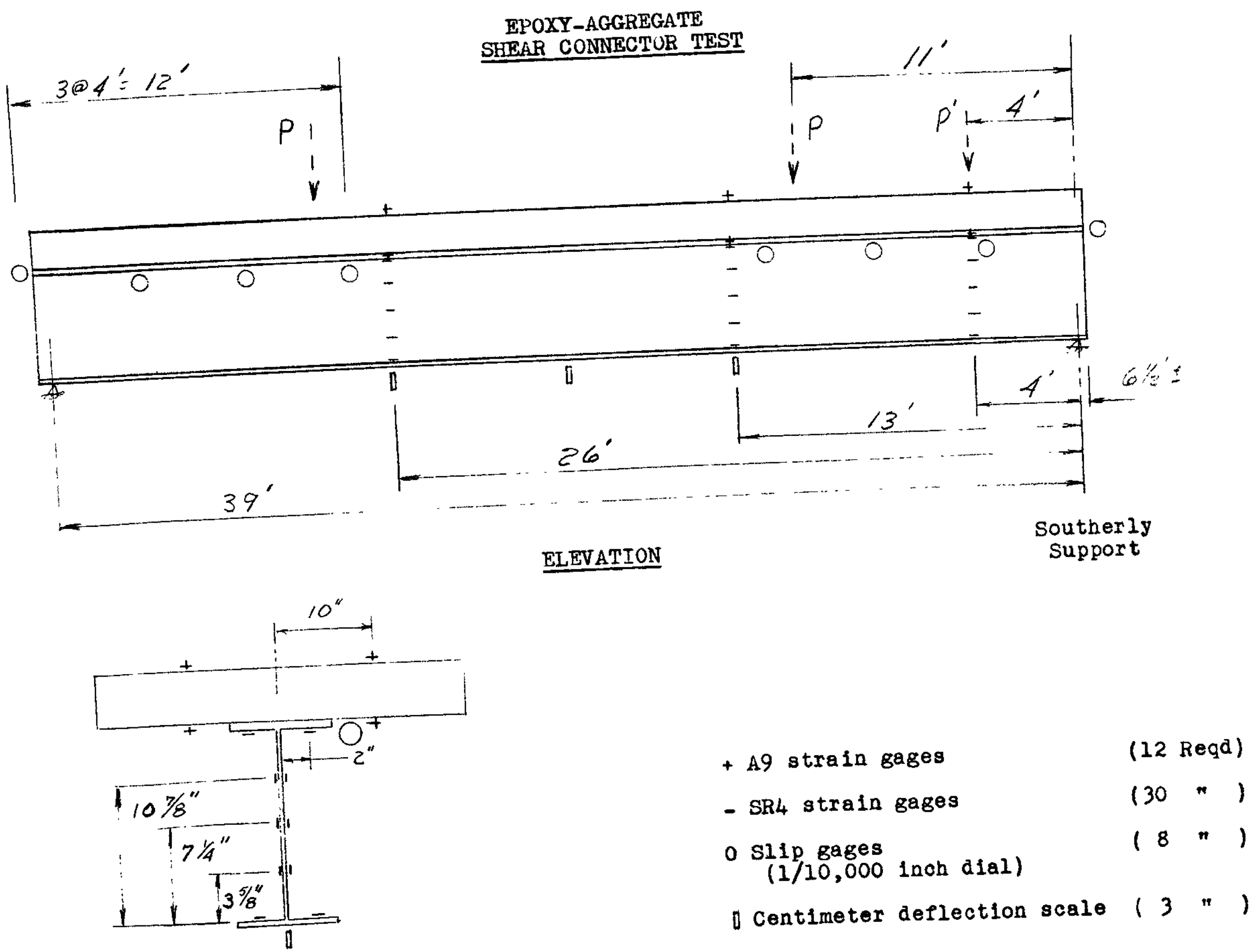
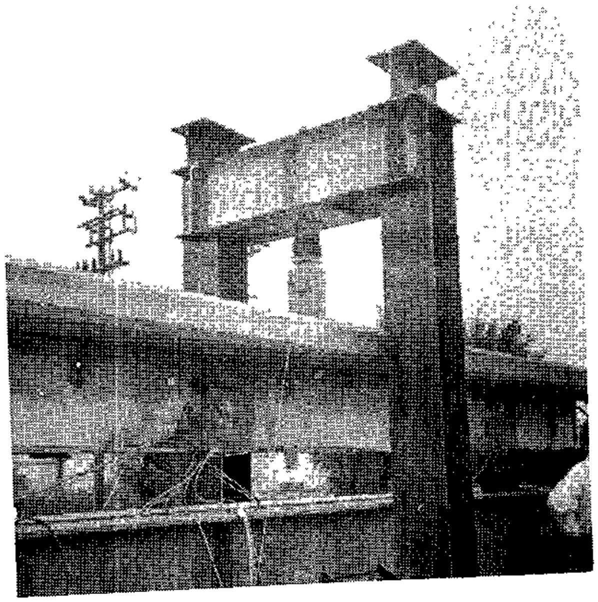
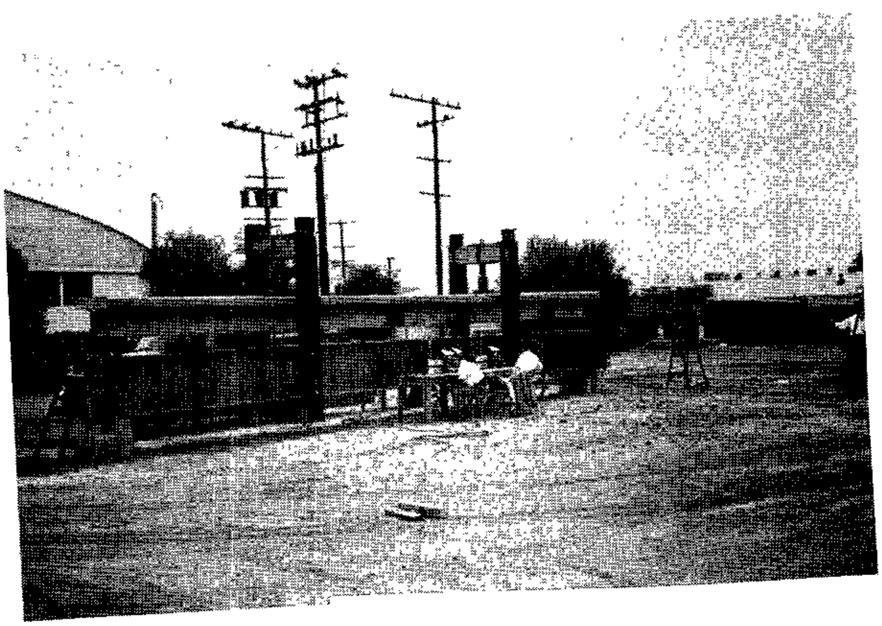


FIG. 2 INSTRUMENTATION

CROSS SECTION

EPOXY-AGGREGATE
SHEAR CONNECTOR TEST



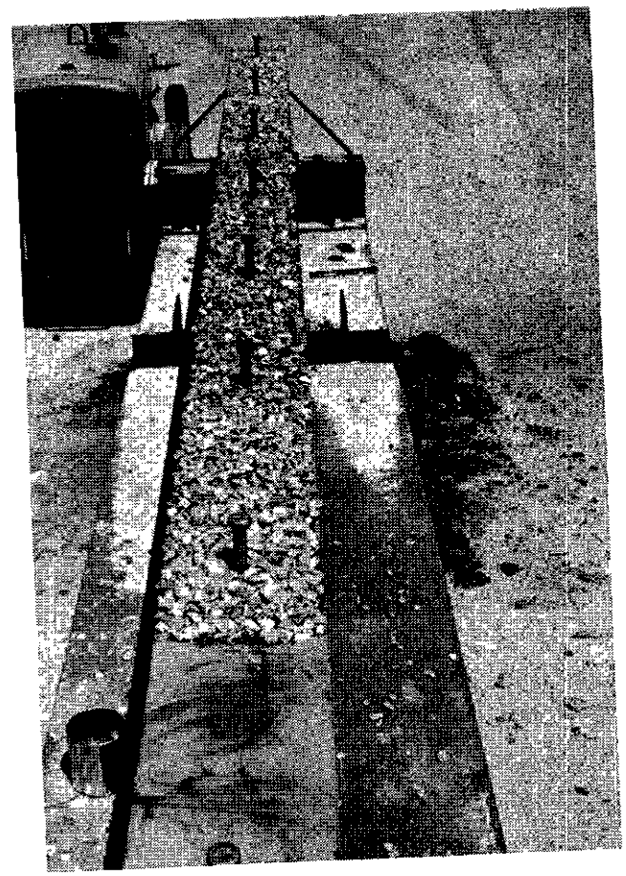


PROTOTYPE TEST GIRDER NO 1.

FIG. 3

EPOXY-AGGREGATE
SHEAR CONNECTOR TEST

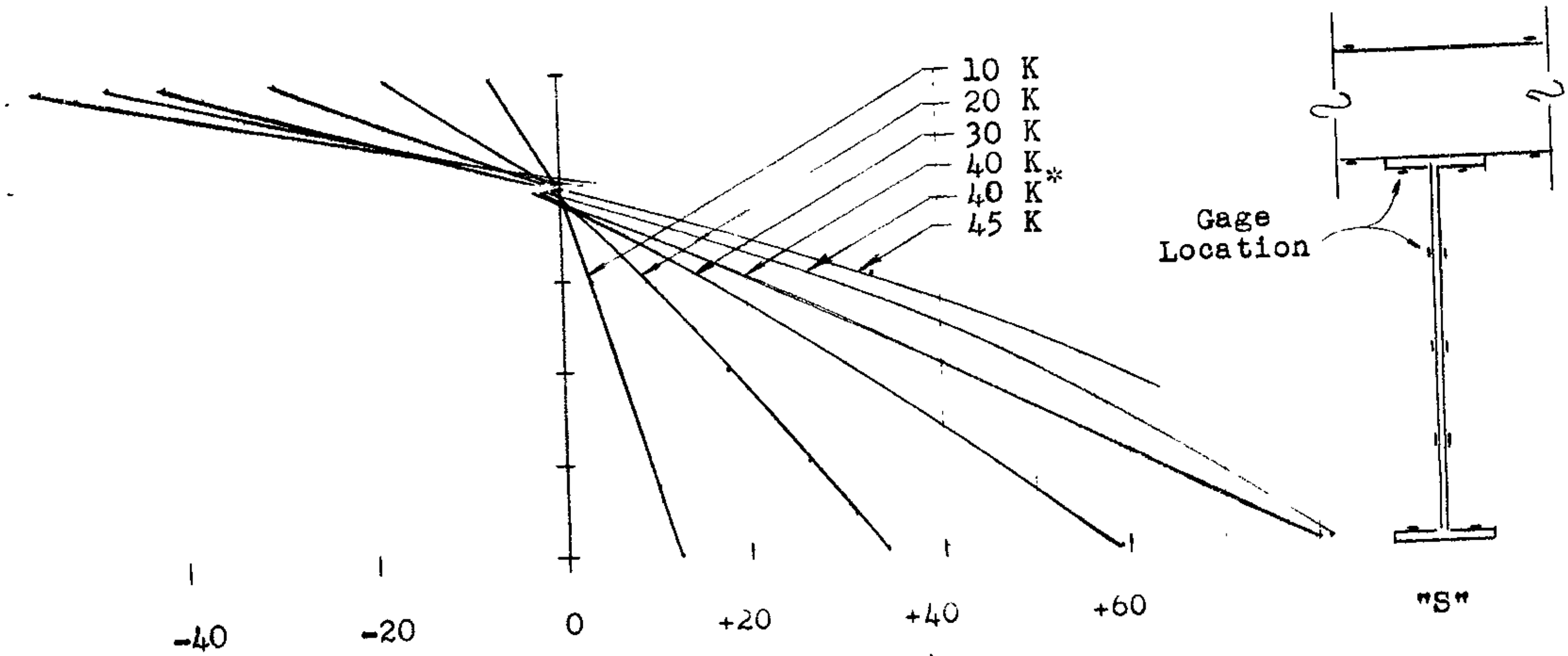
Epoxy-Aggregate
During Application.



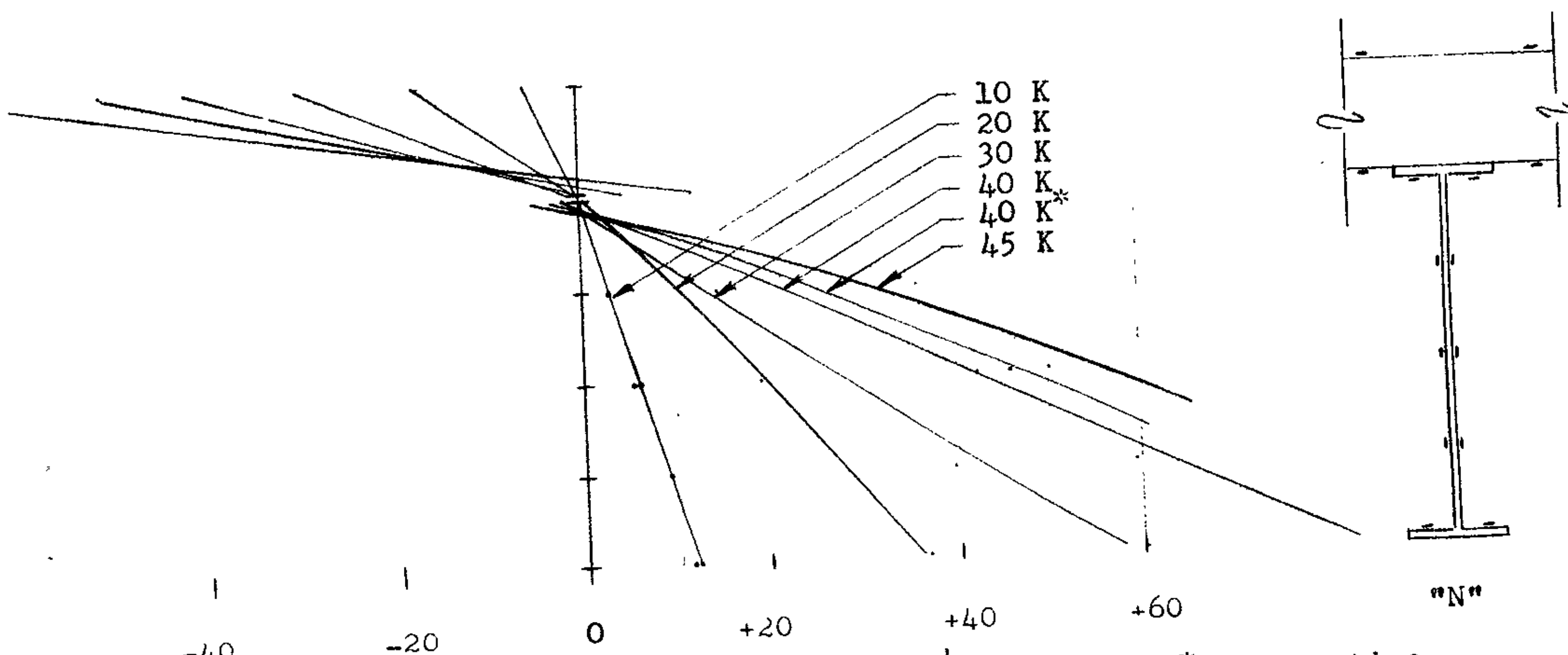
Epoxy-Aggregate
Before Excess Aggregate
Was Removed.

FIG. 4

VERTICAL STRAIN DISTRIBUTION AT 1/3 POINTS
(SYMMETRICAL LOADS NEAR 1/3 POINTS)



STRAIN (in/in x 10^{-4})



STRAIN (in/in x 10^{-4})

*30 Min. time differential

FIG. 5

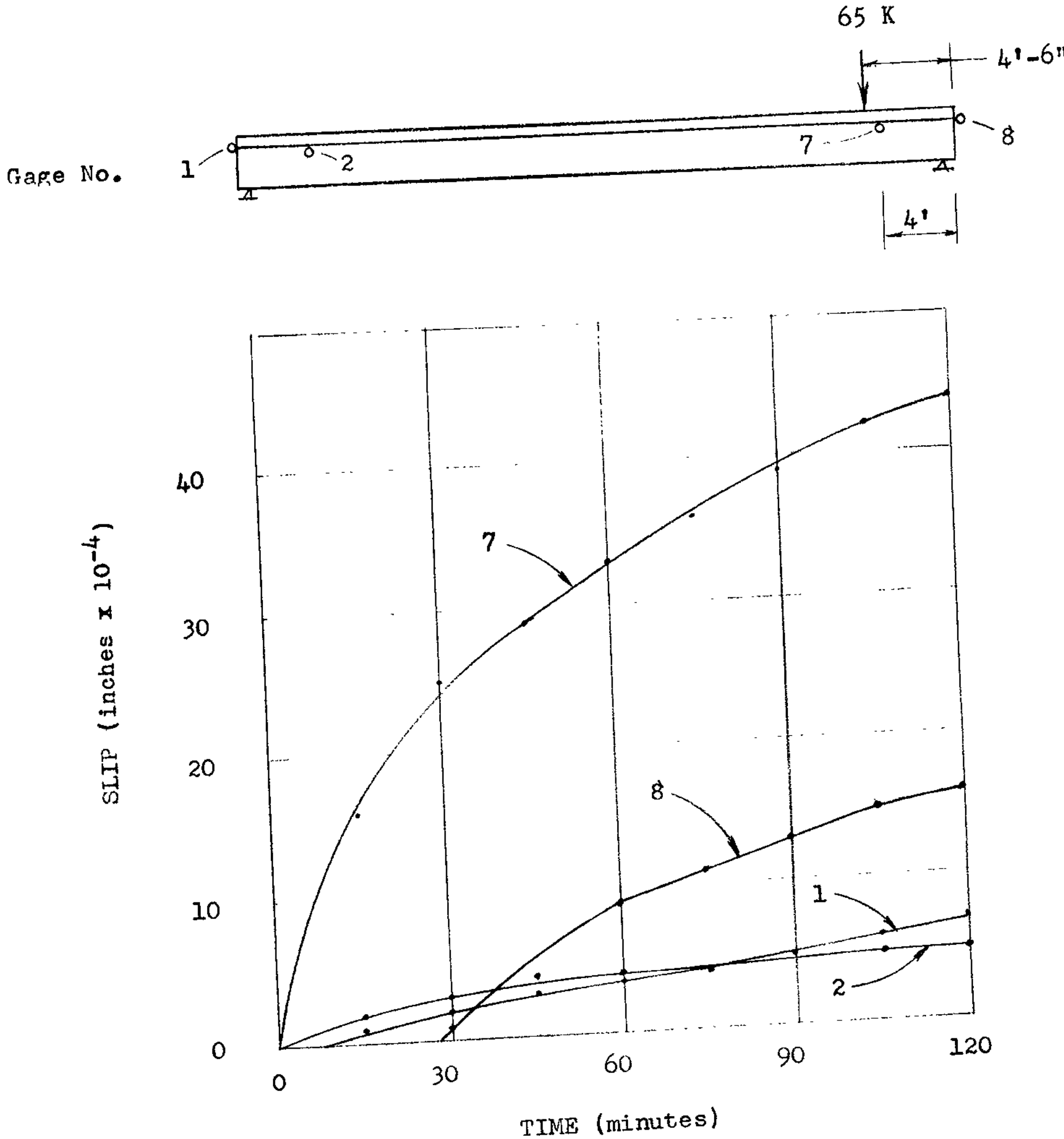
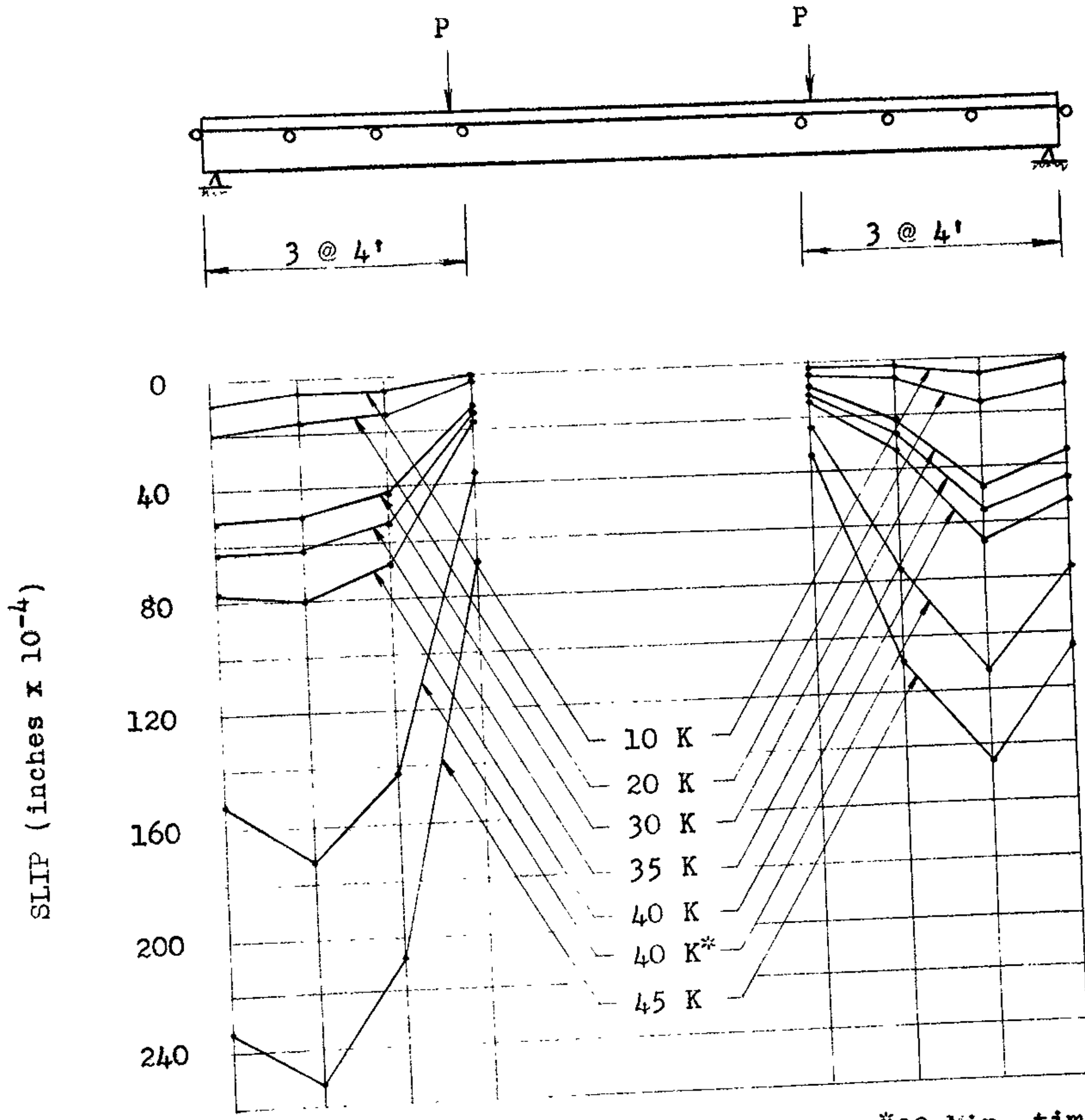


FIG. 6 SLIP-TIME CURVES



*30 Min. time differential

FIG. 7 LOAD-SLIP CURVES

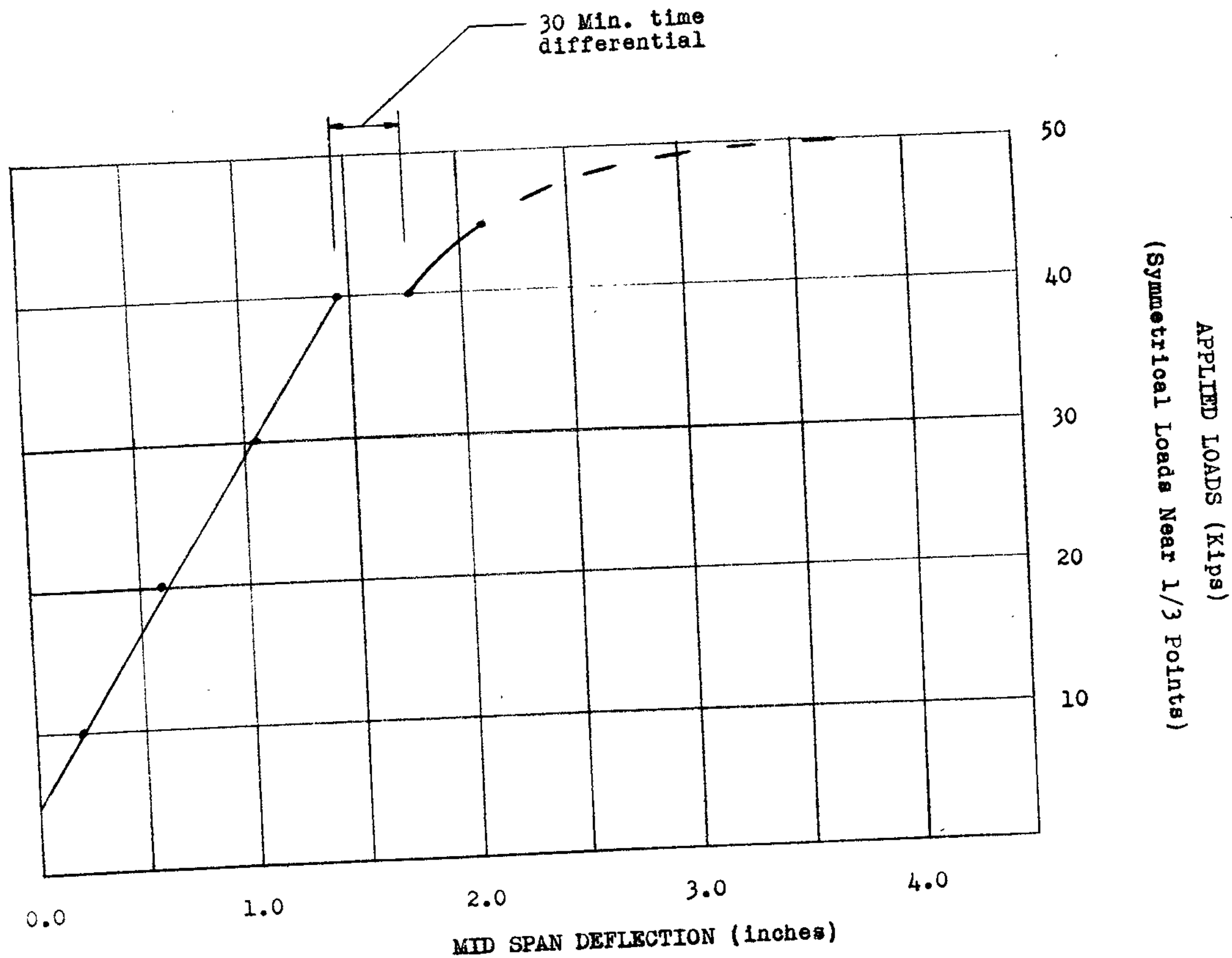


FIG. 8 LOAD-DEFLECTION CURVE
(Two Point Loading)

EPOXY-AGGREGATE
SHEAR CONNECTOR TEST

DOUBLE SHEAR TEST

FORMULATION	CURING TEMPERATURE (F°)	SHEAR (psi)
A	78	16,600
A	140	18,300
B	78	16,800
B	140	19,200
B	140	21,300
B'	78	14,300
B'	140	12,500
B'	140	15,400

FORMULATION DESIGNATION:

A. California Division of Highways - Epon.

B. Arizona Highway Department - Epi-Rez 510.

B'. Formulation B with less fillers.

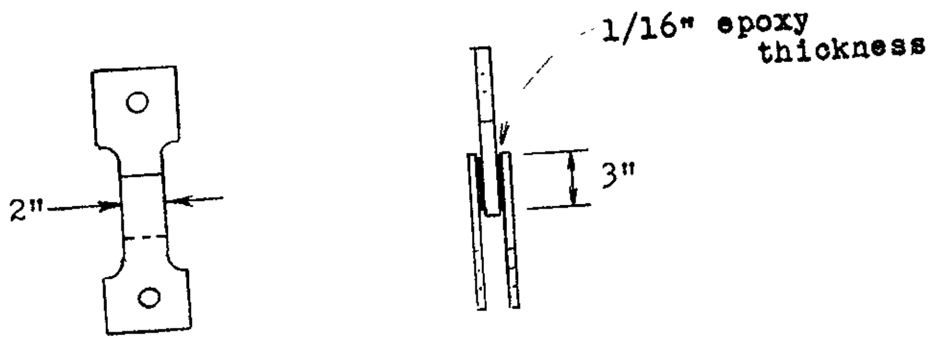
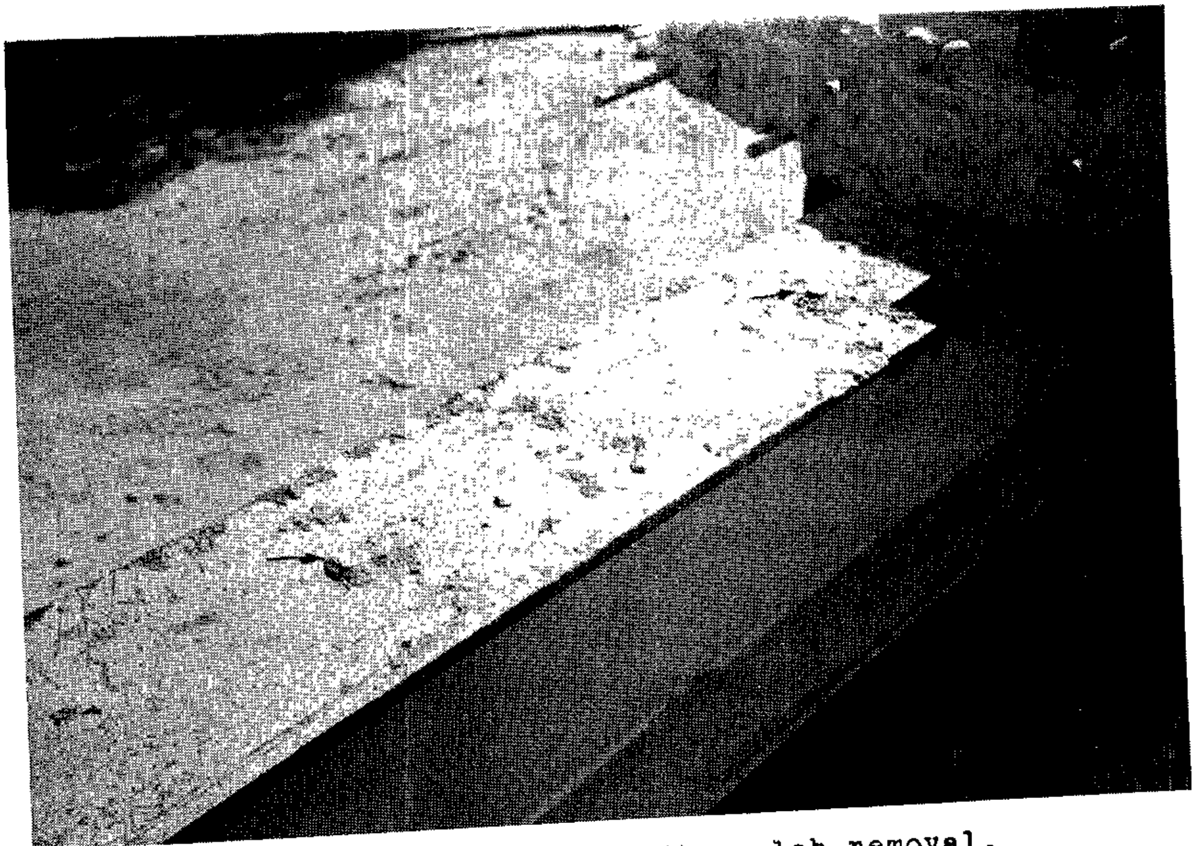


Figure 9

EPOXY-AGGREGATE
SHEAR CONNECTOR TEST



Top of steel girder after slab removal.
Note two sheared steel stud connectors.

Bottom of concrete slab after removal from steel girder.

FIG. 10